COMPDYN 2011
3rd ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis, V. Plevris (eds.)
Corfu, Greece, 25–28 May 2011

SEISMIC ASSESSMENT OF STEEL MOMENT FRAMES USING SIMPLIFIED NONLINEAR MODELS

Dimitrios G. Lignos[1], Christopher Putman[2], and Helmut Krawinkler[3]

[1] McGill University
817 Sherbrooke West Room 278C, Montreal QC, H3A 2K6
e-mail: dimitrios.lignos@mcgill.ca

[2] Degenkolb Engineers
1300 Clay Street, Suite 900, Oakland, CA, 94612, United States
cputman@degenkolb.com

[3] Stanford University
473 Via Ortega, Room 231, Stanford, CA, 94305, United States
e-mail: krawinkler@stanford.edu

Keywords: Simplified modeling, nonlinear static procedures, multi-modal static procedures, gravity system, seismic demands, steel moment resisting frames.

Abstract. *The effectiveness of simplified nonlinear models for seismic assessment of steel moment frames using single and multi-mode nonlinear static methods compared to nonlinear response history analysis is discussed in this paper. Results of studies of different steel archetype buildings with perimeter steel moment resisting frames are compared with those from nonlinear response history analysis (NRHA). Simplified modeling of gravity systems is also discussed. It is demonstrated that the nonlinear static procedure (NSP) has many limitations for quantitative assessment of moment frame demands even for relatively low-rise structures. But it has much value in understanding important behavior characteristics that are not being explored in a NRHA in which engineers usually focus on a "blind" demand/capacity assessment rather than interpretation and visualization of behavior. The conclusion is that both NSP and NRHA have intrinsic value and that it is advisable to employ a combination of both to understand seismic performance and quantify important engineering demand parameters. This study is part of a recent National Institute of Standards and Technology (NIST) funded project through a joint venture partnership of the Applied Technology Council (ATC) and Consortium of Universities for Research in Earthquake Engineering (CUREE) on improvement of nonlinear multiple-degree-of-freedom modeling for design decisions of regular and irregular structural systems (NIST GRC 10-917-9).*

1 INTRODUCTION

Performance Based Earthquake Engineering (PBEE) concepts have been adopted by a number of engineering guidelines such as [1-7] for seismic evaluation and rehabilitation of steel and reinforced concrete structures. A common procedure for a structural engineer to conduct such an evaluation is to utilize nonlinear static procedure (NSP), which is also referred in the literature as pushover analysis. In U.S. practice, a pushover analysis is typically based on an invariant lateral load pattern that is applied along the height of a structure, which is pushed to a pre-defined target roof displacement. Many researchers have conducted extensive research on the evaluation of seismic demands of structural systems with nonlinear static procedures based on invariant load patterns, e.g., [8-12]. In these studies major drawbacks of these procedures to predict seismic demands of structures have been summarized. Others [13-16] have conducted research on enhanced NSPs that account for higher mode effects and either retain the simplicity of invariant load patterns or employ adaptive procedures in which the lateral load pattern varies during the nonlinear analysis. Typically, these methods improve the prediction of engineering demand parameters compared to the single mode PA. The value of NSP is in the fact that this procedure permits inspection of response and is a relatively simple approximate tool to identify critical regions of a structural system in which the potential for significant strength or stiffness discontinuities is relatively high.

In FEMA-440 [5] the major differences between results obtained from nonlinear static and nonlinear response history analysis (NRHA) were attributed to a number of reasons such as the effect of component deterioration on the seismic response of a structural system [6]. Other reasons for differences between NRHA and NSP in prediction of seismic demands of a structural system are (1) P-Delta effects, (2) inaccuracies in the prediction of target roof displacement at which structural response is to be evaluated, and (3) multi-degree-of-freedom (MDOF) effects. In order to improve nonlinear MDOF modeling for structural engineering design practice for better estimation of the seismic response of structural systems such as steel and reinforced concrete moment resisting frames (MRFs) and reinforced concrete and masonry shear walls, the National Institute of Standards and Technology (NIST) initiated a program of focused studies [18]. This paper summarizes one of the analytical studies conducted as part of this program, which addressed the minimum level of MDOF modeling sophistication and appropriateness of nonlinear methods for seismic evaluation of special steel MRFs. This study investigates the effect of higher modes and gravity system as part of the lateral resisting system on engineering demand parameters (EDPs) of steel MRFs such as story drift ratios, story shear forces, overturning moments, residual story drift ratios and absolute floor accelerations along the height of steel MRFs. The investigation is based on simplified nonlinear models of steel MRFs.

2 ARCHETYPE STEEL BUILDINGS

The steel buildings used for evaluation of EDPs predicted with NRHA and NSP procedures are two-, four- and eight-story special MRFs. The structural systems are perimeter moment frame systems of a set of archetype steel buildings designed as part of the NIST [19] project. These structures comprise 3-bay steel MRFs with Reduced Beam Sections (RBS) designed in accordance with AISC 358-05 [20]. A plan view of a typical archetype is shown in Fig. 1. The three steel buildings that are utilized in this study are designed based on Response Spectrum Analysis (RSA) for seismic design category D_{max}. This corresponds to a design spectral acceleration at short period, S_{DS}, and at a period of 1second, S_{D1}, equal to 1.0g and 0.60g, respectively. More details on the design of the archetype steel buildings are summarized in [19, 21]. In the subsequent discussion and figures, the two-, four- and eight-story steel MRFs are denoted as 2-RSA-D_{max}, 4-RSA-D_{max} and 8-RSA-D_{max}, respectively.

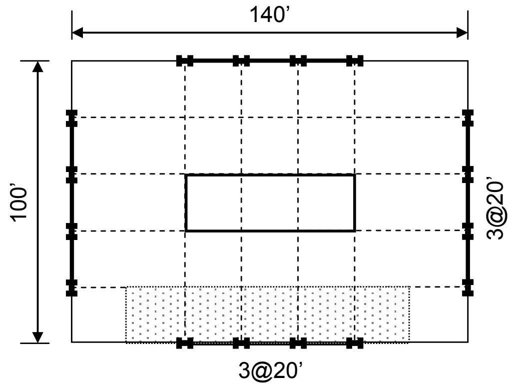


Figure 1: Plan view of typical archetype building with perimeter moment resisting frames.

3 MODELING OF ARCHETYPE STEEL MOMENT RESISTING FRAMES

In order to model the seismic response of the perimeter steel MRFs discussed in Section 2, two analytical models are utilized. Both models are two-dimensional (2-D). In the first analytical model all components (beams, columns and panel zones) of the steel MRF are modeled explicitly. The second model is a single bay simplified frame whose properties are tuned to represent the detailed steel MRF. In both models P-Delta effects are simulated with a leaning column that is connected to the steel MRF with axially rigid links. These links have hinges at their ends. The subsequent sections discuss details of these two analytical models.

3.1 Three-bay steel moment resisting frame model

The steel MRF in the East-West loading direction (see Fig. 1) is modeled in a customized version of DRAIN-2DX (Prakash et al. [22]). This numerical model consists of elastic beam-column elements with concentrated plastic hinge springs at their ends. These springs simulate the hysteretic response a steel component (beam or column) subjected to cyclic loading including strength and stiffness deterioration based on the modified Ibarra-Krawinkler (IK) deterioration model [23, 24]. Panel zones are modeled with the model discussed in [25], which explicitly represents panel zone shear distortions including the possibility of nonlinear behavior during an earthquake. The exact location of the RBS section is also incorporated in the model. The deterioration parameters of beams with RBS and steel columns are determined by multivariate regression equations that have been developed based on information retrieved from a recently developed database for deterioration modeling of steel components [24, 26]. These analytical models were used extensively for quantification of building seismic performance factors using the FEMA P-695 methodology [19].

3.2 Simplified single-bay frame model

To reduce the computational effort in estimating seismic demands of the steel MRFs with NRHA and evaluate the effectiveness of simpler representations of the 3-bay steel MRFs discussed in Section 3.1, a simplified model as shown in Fig. 2 is developed. In this model a single bay frame in a manner represents the three-bay moment-resisting frame so that overturning moment and column axial deformation effects are adequately represented. Luco et al. [27] developed similar models for computing the seismic inelastic demands of steel MRFs. P-Delta effects are simulated with a leaning column that is always present in the numerical model. Strength and stiffness properties of the gravity framing that is not part of the moment resisting frame can be represented with the fishbone model shown at the right of Fig. 2. Lumping together multi-bay frames into a single bay frame can be accomplished by the following rules:

$$\sum EI_i/L_i = EI/L \tag{1}$$

$$\sum M_{p,i} = M_p \tag{2}$$

in which I_i and L_i is the moment of inertia and length of the i-th beam in a story, respectively, and EI/L and M_p are the stiffness and plastic moment of the single bay beam. For steel columns,

$$\sum EI_i = 2EI \tag{3}$$

$$\sum M_{pc,i} = 2M_{pc} \tag{4}$$

in which $M_{pc,i}$ is the plastic moment of the i –th column of the multi-bay frame and M_{pc} is the plastic moment of the single bay column in the presence of an axial load. For taller steel MRFs in which overturning moment and axial deformations in columns are important, these effects can be approximated by setting L of the single bay frame equal to the distance between end columns of the multi-bay frame, and setting the area of the single bay column equal to the area of the end column of the multi-bay frame. This simplification is based on the assumption that overturning effects are resisted mostly by the exterior columns of a steel MRF. The approximations summarized herein are reasonable if all bays of the steel MRFs are of about equal width, and become more approximate when spans of the steel MRF vary considerably. Ignoring panel zone shear deformations and using centerline dimensions for beams and columns introduce additional approximations to the analytical models.

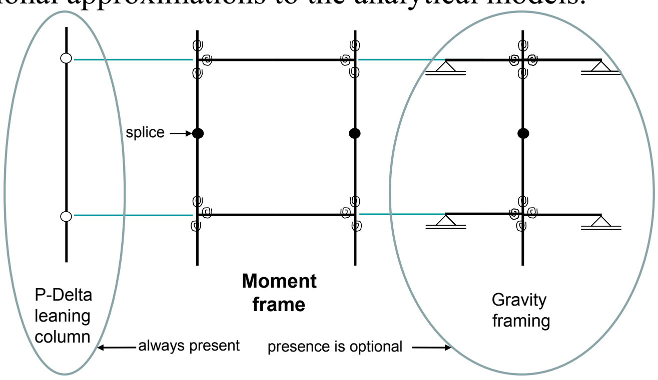
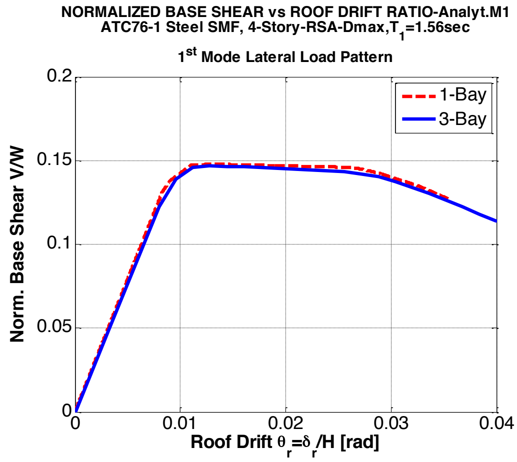
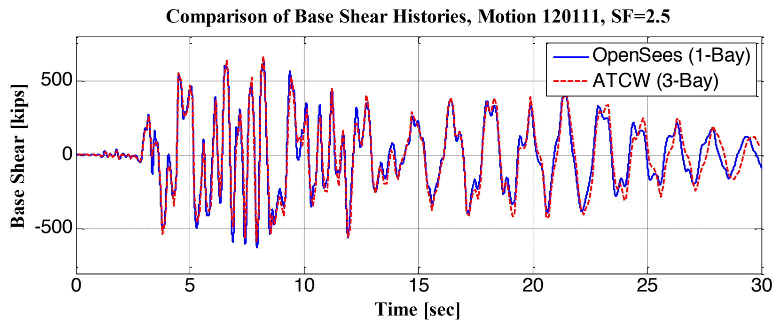


Figure 2: Simplified model with P-Delta column and gravity framing fishbone model.

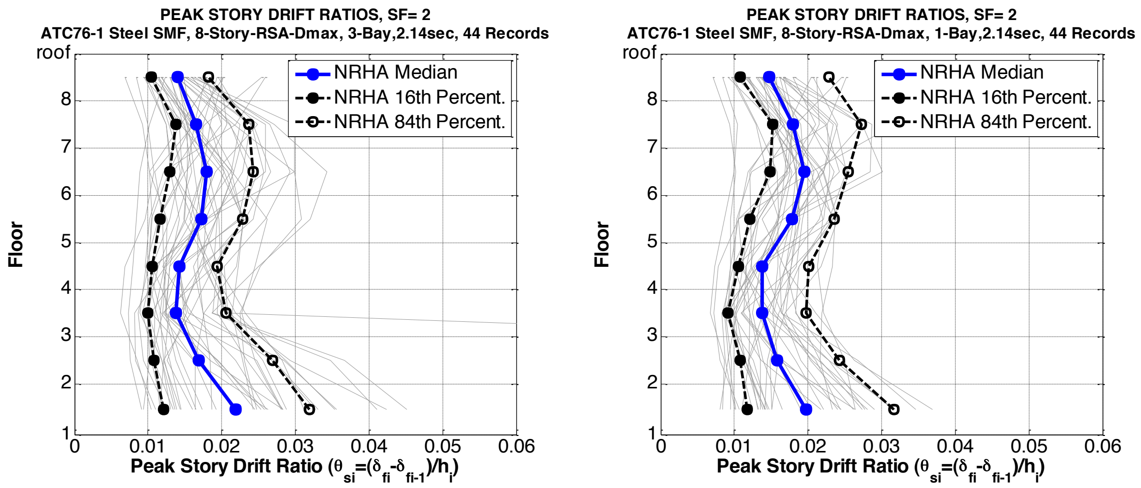
Figure 3a shows a comparison between the pushover curves of the 3-bay four-story steel MRF and the simplified 1-bay model. In this figure, the base shear V is normalized with respect to the seismic weight W of the steel MRF. The base shear V is computed from the inertia forces only (V_I). The roof drift θ_r is defined as roof displacement δ_r over the total height H of the steel MRF. As seen from this figure, the response of the 1- and 3-bay models is almost identical. The 1-bay models are implemented in the OpenSees [28] simulation platform whereas the 3-bay models are implemented in Drain-2DX [22]. The base shear histories for a single ground motion obtained from the 3-bay model developed in Drain-2DX (ATCW 3-Bay) and the 1-bay model developed in OpenSees are shown in Fig. 3b. . Simulations are carried out for the FEMA P-695 [7] set of 44 ground motions for three scale factors (0.5, 1.0 and 2.0). Note that a scale factor SF=2.0 corresponds to approximately a maximum considered event (MCE) in California. A comparison between absolute peak overturning moments (OTM) obtained for the eight-story 3-bay and 1-bay frames is shown in Fig. 3c. In both static and dynamic analysis the seismic response based on the 1-bay and 3-bay models is almost identical, providing confidence in both the simulation platforms and in the ability of the simplified model to represent the response of the 3-bay steel MRF.



(a) Comparison of pushover curves between 1-bay and 3-bay models of the four-story steel MRF



(b) Base shear history of the four-story MRF models for a single ground motion



(c) Comparison of overturning moments obtained from NRHA of 3-bay and simplified 1-bay model

Figure 3: Comparison of response predictions using the 3-bay and 1-bay simulation models.

4 SEISMIC ASSESSMENT OF STEEL MOMENT RESISTING FRAMES

This section focuses on evaluating the seismic response of steel MRFs with simplified models discussed in Section 3.2. The assessment is based on a comparison between NSP and NRHA results. Feasibility and limitations of the NSP is illustrated with the case studies that were investigated. Since the emphasis is on simple methods that can assess multi-mode effects on the seismic response of steel frame structures the modal pushover analysis (MPA) [13] is also evaluated. Two main options are used for modeling the components of the steel MRFs discussed in Section 3. These options are summarized as follows:

- **ASCE41:** all the steel components are modeled in accordance with ASCE/SEI 41-06 [3] utilizing the component model shown in Fig. 4a. Note that a post-capping stiffness obtained by linearly connecting peak point C and point E of the generic ASCE/SEI 41-06 model is used. This modification is made in order to provide a better match with data and analysis models developed in the past decade [23, 24] and also to avoid numerical stability problems in the analysis.
- **Analyt.M1:** all the steel components are modeled with the modified IK component model [24]. For this purpose, a monotonic backbone curve is used as shown in Fig. 4b. This option is the same as the ATC-72-1 [29] analysis Option 1. Cyclic deterioration is not reflected in the component model for monotonic response and subsequently in the NSP. However, in the NRHA the component model deteriorates cyclically based on rules discussed in [23, 24]. For comparison purposes in the same figure we have superimposed the modified backbone curve based on the IK model (see Fig. 4b) based on the ATC-72 [29] analysis option 3.

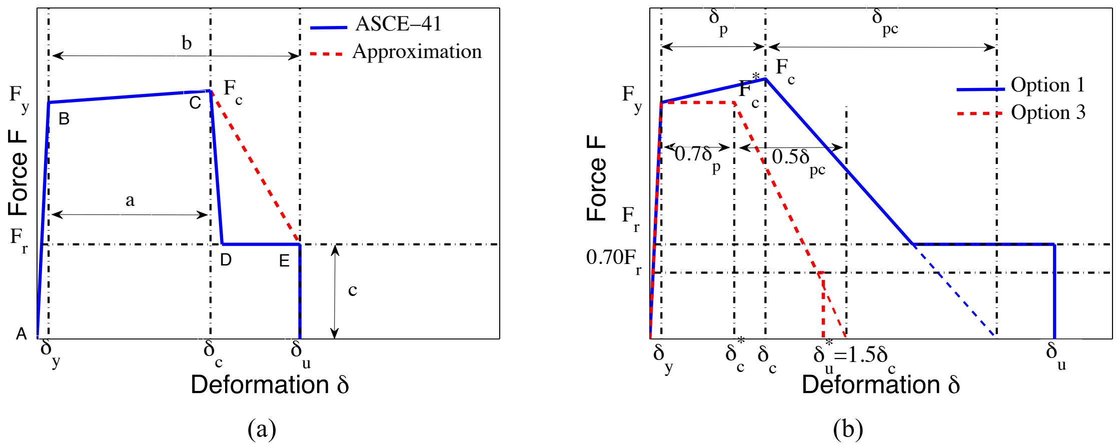


Figure 4: Steel component models; (a) ASCE 41; (b) Modified IK model, Options 1 and 3.

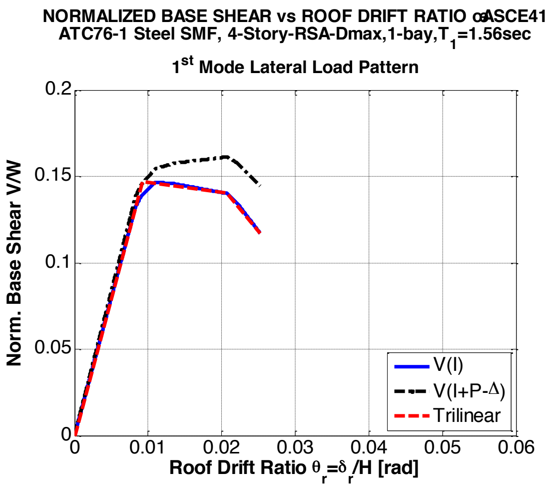
The following two options are used to determine the target roof displacement for the NSP:

- **ASCE41**: Target displacement obtained from ASCE/SEI 41-06 [3] coefficient method.
- **EqSDOF:** Target displacement based on median displacement obtained from NRHA of the first mode equivalent Single-Degree-Of-Freedom (SDOF) system using the 44 ground motions of the FEMA P-695 [7] set and the analysis tool IIIDAP (Lignos [30]). Equivalent SDOF properties are obtained from the base shear without P-Delta V_I – roof displacement pushover curve (not the base shear including P-Delta $V_{I+P\text{-}\Delta}$ – roof displacement pushover curve), which implies that P-delta effects are accounted for approximately in the properties of the equivalent SDOF system.

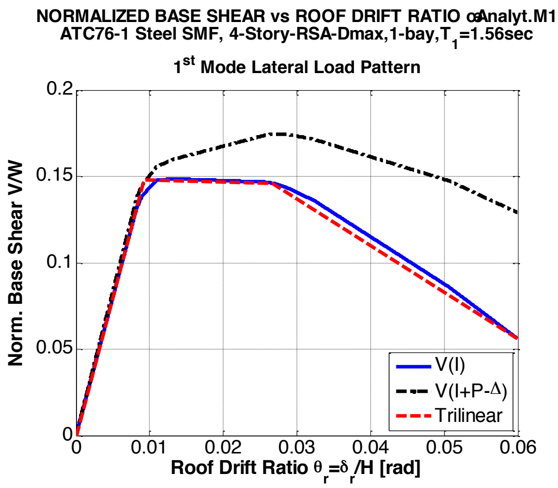
Note that for the MPA procedure only the Analyt.M1-EqSDOF option was explored. This implies that the pushover analysis is conducted with the Analyt.M1 model, and the target displacement for the individual modes is determined from an equivalent SDOF analysis with the analysis tool IIIDAP [30], which computes inelastic response of SDOF systems with due consideration given to deterioration. The cyclic deterioration parameter λ [26] is set equal to the median value for steel components obtained from a steel database for deterioration modeling (Lignos and Krawinkler [26]).

4.1 Single mode nonlinear static analysis procedure

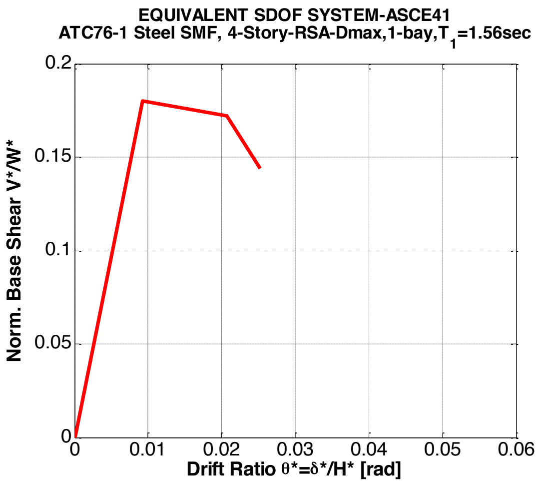
Figures 5a and 5b show the pushover curves with ($V_{I+P\text{-}\Delta}$) and without P-Delta (V_I) effects for the four-story steel MRF based on the ASCE-41 and Analyt.M1 component models, respectively. In the same figures we have superimposed the idealized trilinear curve based on the ASCE-41-06 [3] criteria. These figures show that the NSP based on the ASCE41 component model underestimates the post-yield strength and deformation capacities compared to the Analyt.M1 model. The implication is that the target roof displacements predicted from the equivalent SDOF systems shown in Fig. 5c and 5d are different for large ground motion demands.



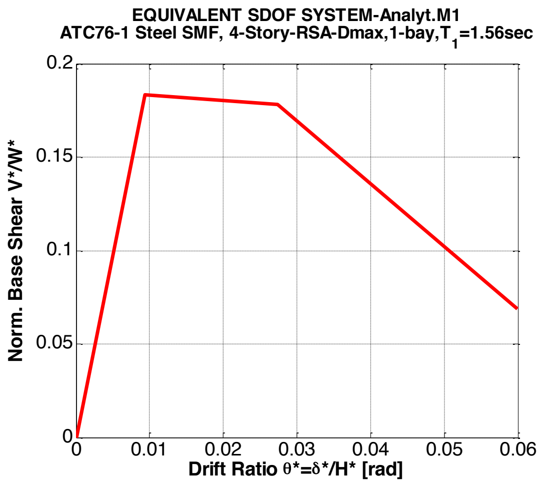
(a) ASCE-41 component model



(b) Analyt.M1 component model



(c) ASCE41 component model



(d) Analyt.M1 component model

Figure 5: Single mode pushover curves and equivalent SDOF systems for the 4-story-RSA-Dmax steel MRF.

The use of the pushover curve based on the ASCE-41 component model (see Fig. 5a) together with the equivalent SDOF model for target displacement prediction (ASCE41-EqSDOF) may provide performance estimates that are lower than might be justifiable. For a scale factor SF = 2.0 the EqSDOF leads to 33 collapses, which are a direct consequence of the relatively short yield plateau obtained from using ASCE41 component models in the pushover analysis. For all options, NSP story drift predictions show a significant deviation from median NRHA values (Fig. 6a, 6b). In the inelastic range (SF = 2.0) drifts in the lower stories are overestimated and drifts in the upper stories are underestimated. Results are illustrated for the two- and four-story steel MRFs but the same observation applies for the eight-story MRF.

Nonlinear static procedure story shear predictions for the four-story steel MRF show poor correlation with NRHA results in the inelastic range (SF = 2.0). This can be seen in Fig. 6c and 6d. Story shears are consistently underestimated, particularly in the upper stories. The reason is dynamic redistribution, which amplifies story shear forces compared to those obtained from a predetermined lateral load pattern. If story shears are an important performance consideration, then the validity of quantitative values obtained from a pushover analysis diminished for this 4-story steel SMF. Similar observations apply to floor OTMs, which control axial forces in columns of frame structures. In the upper stories, the NSP predictions are less than half those obtained from NRHA (see Fig. 6e and 6f). The situation is better at the base, because absolute maximum shear forces in individual stories occur at different times. The outcome is that even for relatively low-rise steel MRF structures NSP predictions may provide misleading quantitative information, particularly for force quantities.

The all-important issue of lateral load pattern is not explored here. Previous work [5] has addressed this issue and came to the conclusion that variations in invariant lateral load patterns do not improve the accuracy of EDP predictions. The load pattern applied in all cases discussed here is the pattern structured after the elastic first mode deflected shape, as recommended in [3].

4.2 Incorporation of gravity system in analysis model

Gravity system components must have sufficient strength and deformation capacity to resist tributary gravity loads at the maximum drifts computed for the lateral load resisting system. The structural engineer typically decides weather or not to include contributions of the gravity system to lateral stiffness and strength of a building. It is recommended to incorporate the gravity system in the analytical model of the structural system because the analysis might expose weaknesses that are not evident from inspection. An incentive for incorporating the gravity system is its potential benefit in decreasing drift demands and increasing collapse capacity. This might be particularly attractive if the pushover curve exhibits an early negative tangent stiffness that may lead to large displacement amplification or even collapse. The negative stiffness will be reduced potentially by incorporating the gravity system or might even turn into a positive stiffness (Gupta and Krawinkler [31]).

A simple way to incorporate the gravity framing is by means of the “fishbone” arrangement shown earlier in Fig. 2. In order to prevent accumulation of large axial force in the column of the fishbone, an arrangement with two beams is preferred. In this arrangement all beams are lumped into a single beam (I/L of beam = $\Sigma EI_i/L_i$ of all beams), all columns are lumped into a single column (I of column = ΣI_j of all columns), and all gravity connections are lumped into two connections represented by rotational springs.

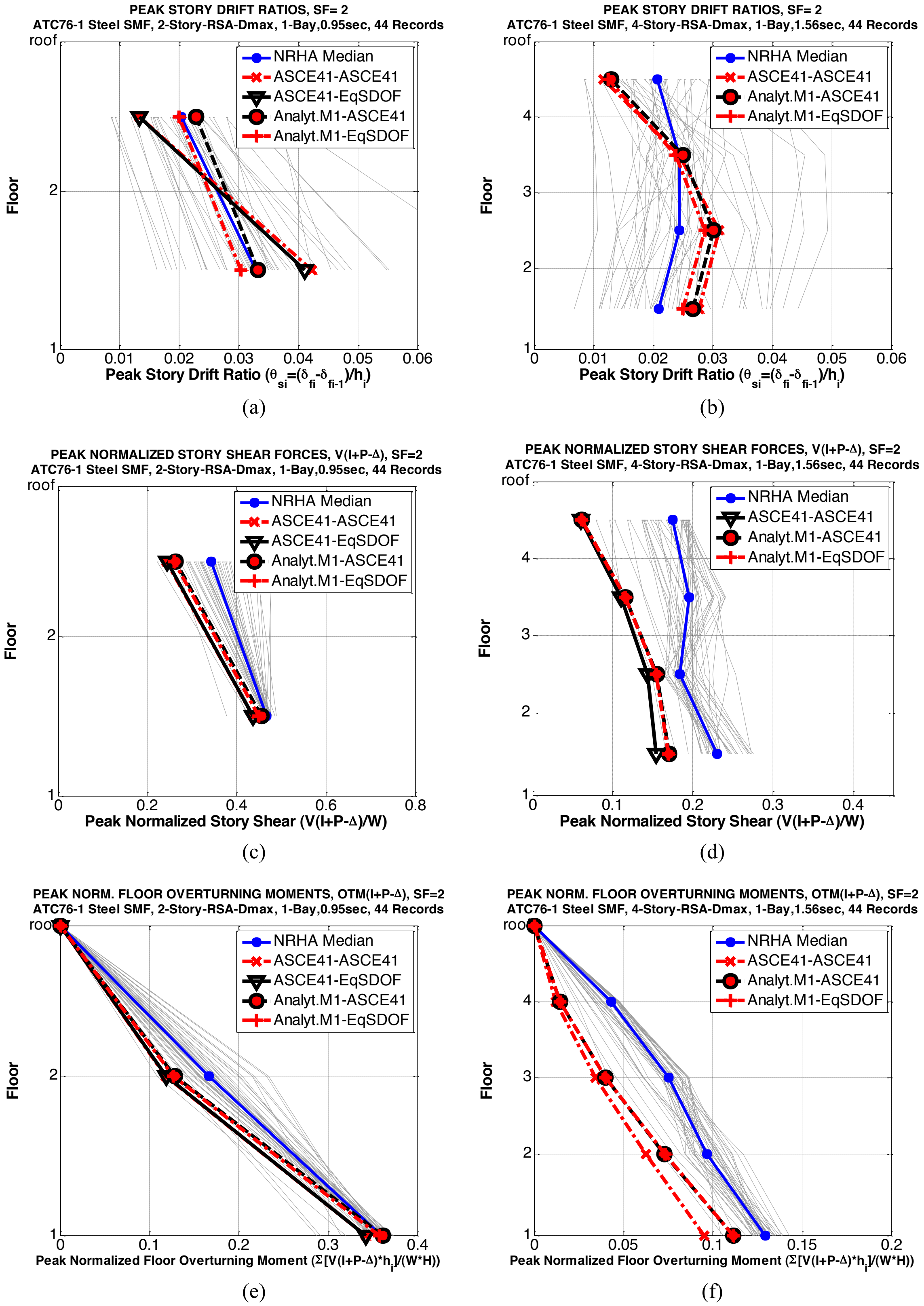
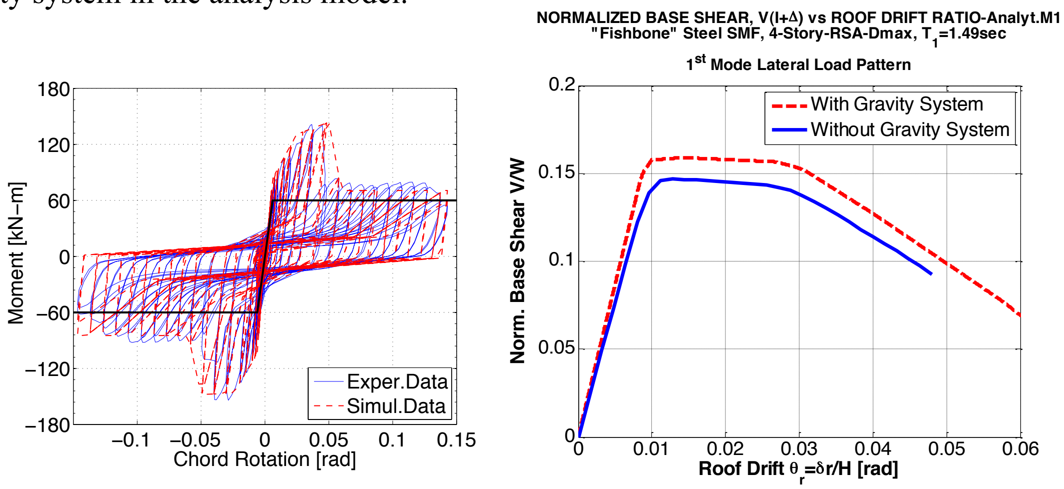


Figure 6: Comparison between NSP and NRHA predictions for EDPs of the two and four-story steel MRFs for SF=2.0.

Beams can be represented usually by elastic elements, provided the connections of beams to columns are weaker than the beams. Column bending strength should include the effects of tributary axial forces due to gravity loads. Post-yield properties of the columns should be based on average plastic hinge properties of the column sections. For modeling of the 4-RSA-D_{max} gravity system, a preliminary design of the gravity beams and columns is performed using tributary areas deduced from the plan view shown in Fig. 1. Since only half of the structure is modeled, the spine (column) of the "fishbone" represents 6 gravity columns and four moment frame columns bending about the weak axis. The beam represents 7 gravity beams.

Connection properties were estimated from tests summarized in Liu and Astaneh-Asl [32, 33]. The cyclic behavior of a typical steel shear tab connection is shown in Fig. 7a. From this figure, the hysteretic response of the gravity connection is pinched. Utilizing the pinching04 model in OpenSees [28] the simulated response of this connection matches the experimental results fairly accurately. However, because of the complex behavior of these connections, greatly simplified and generally conservative models can also be employed that are easily utilized by the engineering profession; thus, a simple elastic-perfectly plastic spring model superimposed on the experimental results is also used. The yield rotation for this spring is 0.008, which is about the same as the yield rotations of the beams of the steel MRF. Pre-capping plastic rotation θ_p is 0.10 and post-capping θ_{pc} is assumed as 0.15. The yield strength is a compromise between positive and negative strength values that can be sustained at very large inelastic rotations. This model ignores the additional strength at relatively small rotations. A comparison of pushovers without and with gravity system is presented in Fig. 7b. In this example not much is gained in pushover strength and deformation capacity by incorporating the gravity system in the analysis model.

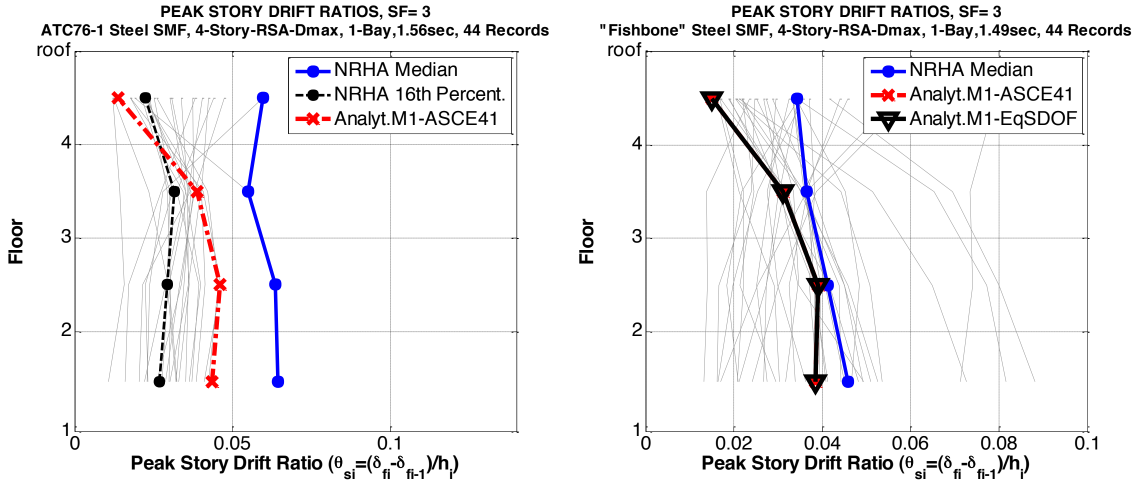


(a) Hysteretic response of a shear connection (data from [32,33])

(b) Comparison of pushover curves of four-story steel MRF with/without gravity system

Figure 7: Effect of gravity system on global pushover of the four-story steel MRF.

The gain in peak story drift ratios when incorporating the gravity system in this example is seen in Fig. 8, which show NRHA and NSP results for a ground motion scale factor SF = 3.0. For this large ground motion scale factor the maximum response is mostly in the negative tangent stiffness region of the pushover (roof drift > 3% as seen from Fig. 7b). Figure 8a shows the median story drift ratios along the height of the bare four-story steel MRF for the set of 44 ground motions for SF=3.0. This scale factor represents the median collapse capacity of this steel MRF, because collapse occurred under 22 of 44 ground motions. Incorporation of the gravity system reduced the number of collapses from 22 to 11, which has a significant effect on the probability of collapse. The median roof drift is reduced from 0.049 to 0.034, and the effect of the gravity system on peak story drifts can be inspected from the figure. The observations made here are case specific, and the benefit gained from incorporating the gravity system may depend strongly on the structural configuration.



(a) Bare steel MRF (b) Steel MRF with gravity system

Figure 8: Effect of gravity system on story drift ratios of the four-story MRF for SF=3.0.

4.3 Multi mode nonlinear static procedures

In this section, the assessment of the four- and eight-story steel MRF based on the MPA procedure is discussed. The basic steps for seismic evaluation of the peak response of a structural system using MPA are summarized in [13,18]. Figures 9a and 9b show the pushover curves of the four-story steel MRF using the 1st and 2nd mode lateral load pattern. The idealized equivalent SDOF systems based on the Analyt.M1 component model are shown in Fig. 9c and 9d for the 1st and 2nd mode load pattern, respectively. The IIIDAP program was used to compute median displacements for the equivalent modal SDOF systems using the 44 FEMA P695 ground motions.

Based on Goel and Chopra [34] an improved estimate of plastic hinge rotations and member forces using MPA into the inelastic range can be obtained by computing plastic hinge rotations from the total story drifts [36]. However, this will require an additional nonlinear static analysis. For simplicity purposes, this approach was not implemented in the results presented in this section. But in many cases, particularly for low-rise regular structure, the higher mode target displacement obtained from the equivalent SDOF system is less than the yield displacement, which implies that the higher mode contribution is elastic. If this is the case, all deformations and forces obtained from the MPA are modal combinations of inelastic first mode contributions and elastic higher mode contributions. In general, this is a preferred procedure compared to the elastic response spectrum analysis (RSA) in which all modal contributions are assumed to be elastic up front [18].

The results presented in this section are for the four- and eight-story steel MRFs. Note that their seismic response has not entered the negative tangent stiffness region. The following summary observations are made on the benefits of MPA predictions for steel SMFs compared to single mode NSP predictions:

1. In all cases investigated the MPA led to improved EDP predictions compared to the single mode NSP options. The MPA employed here is based on the component model used in the NRHA (Analyt.M1) and on predicting modal target displacements from NRHA of equivalent modal SDOF systems.

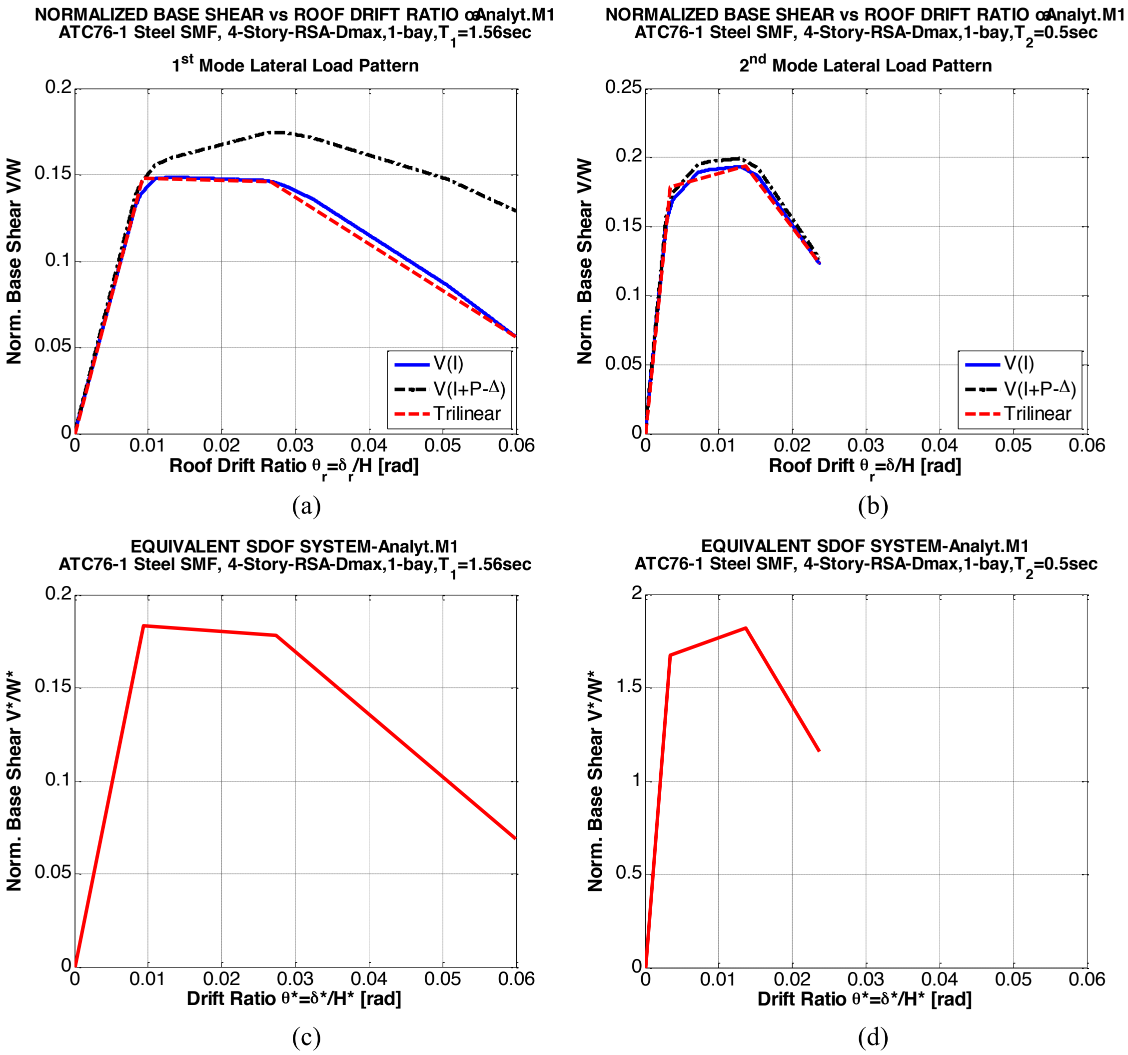
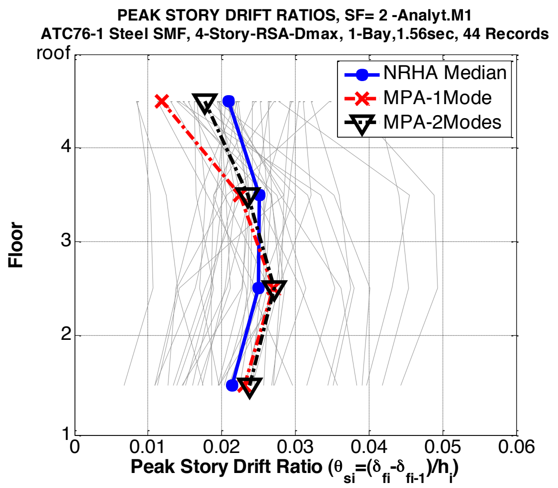
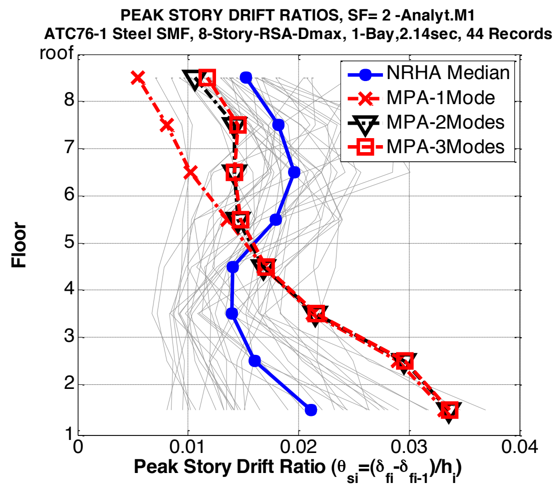


Figure 9: Pushover curves for 1st and 2nd lateral load pattern of the 4-story-RSA-Dmax steel MRF together with equivalent SDOFs.

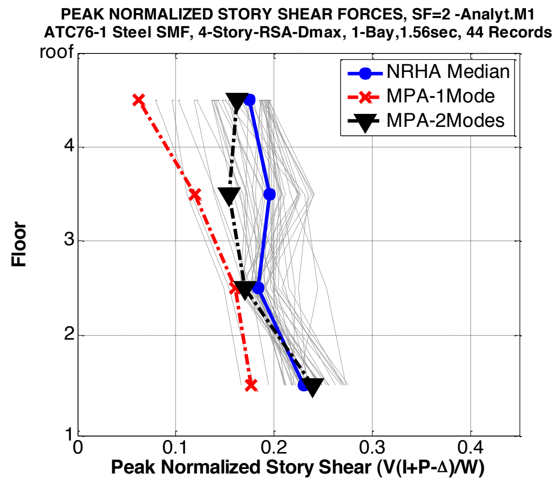
2. Incorporation of the second mode led to considerable improvement in EDP predictions. Consideration of the 3rd mode did not change the results by much even for the eight-story steel MRF (see Fig. 10).
3. In the case of the four-story steel MRF (see Fig. 10a, 11c, 10e) the improvement of all story-based EDP predictions compared to NSP predictions is remarkable. In the eight-story steel MRF the MPA significantly improved story drift ratios (Fig. 10b), shear force (Fig. 10d), and overturning moment predictions in the upper stories (Fig. 10f), compared to NSP. But predicted drifts based on MPA in the lower stories are more than 50% larger than those obtained from NRHA for a ground motion scale factor of 2.0. The reason is that for this scale factor the first mode pushover shows large amplification of story drifts in the lower stories, which is not present in the NRHA. This shows the sensitivity to invariant load patterns, which is present as much in the MPA as it is in a single mode NSP.
4. The second mode contribution was elastic, which simplifies the modal combination and avoids ambiguities that might be caused by displacement reversals sometimes observed in inelastic higher mode pushover analyses.



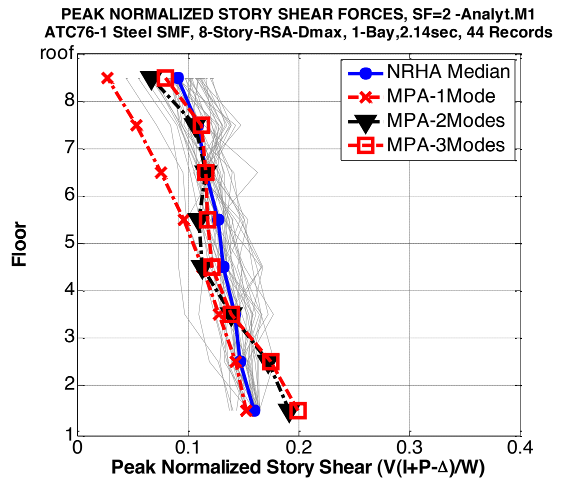
(a) four-story steel MRF



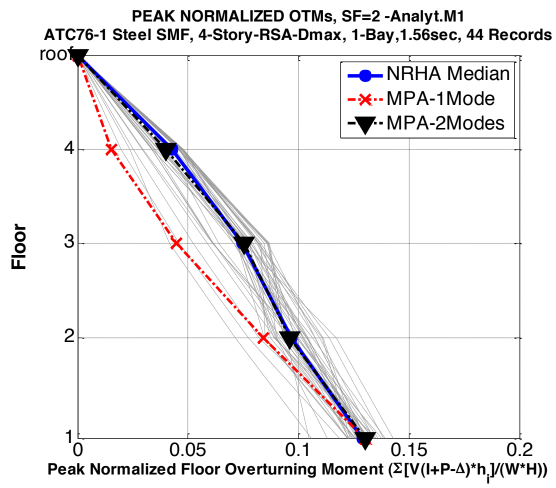
(b) eight-story steel MRF



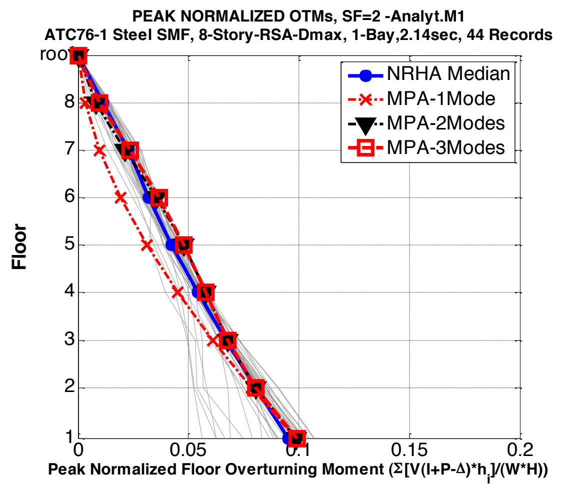
(c) four-story steel MRF



(d) eight-story steel MRF



(e) four-story steel MRF



(f) eight-story steel MRF

Figure 10: Peak EDPs for the four and eight-story steel MRF as predicted with NRHA and MPA for SF=2.0.

4.4 Residual deformations and absolute acceleration demands

Figure 11 shows residual story drift ratios and peak absolute floor accelerations along the height of the four-story steel MRF for SF=1.0 and 2.0. It is noteworthy that the maximum absolute floor acceleration does not vary radically over the height of the four-story MRF, and that it is distributed almost uniformly over the height for a ground motion scale factor of 2.0 at which the structure responds in the highly inelastic range. Similar observations are made for all structures in this study. It is a shortcoming of the NSP that it does not provide any estimation of these two important EDPs, considering the increasing importance of floor acceleration and residual drift in loss assessment of structures [37-39], and the importance of floor accelerations in estimating diaphragm forces.

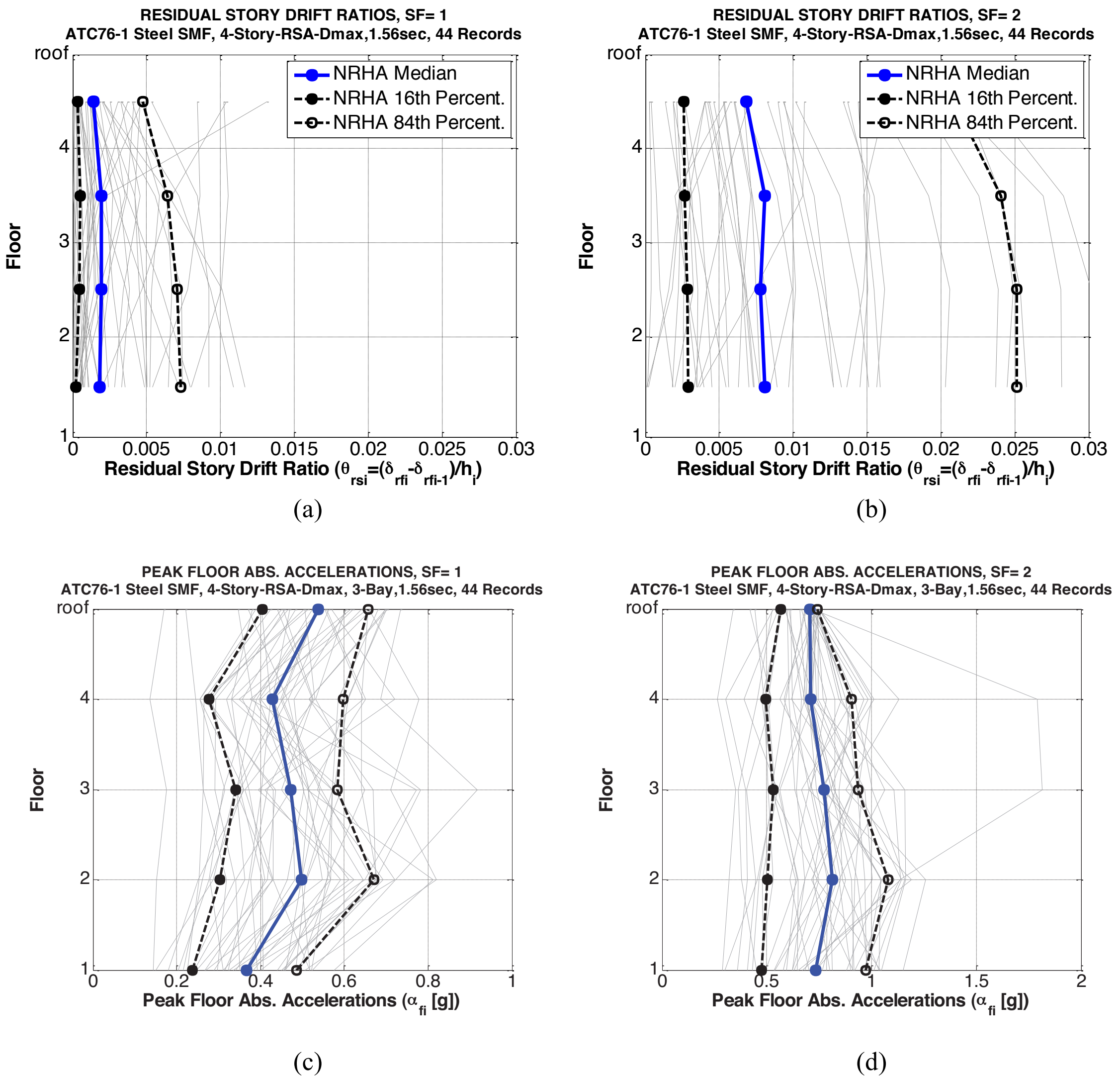


Figure 11: Residual story drift ratios and peak floor accelerations for the four-story MRF for SF=1.0 and 2.0.

5 CONCLUSIONS

This paper presents an assessment of simplified techniques for the seismic evaluation of steel moment resisting frames. This assessment is based on direct comparison of engineering demand parameters such as story drift ratios, story shear forces and overturning moments as predicted with nonlinear single and multi mode static procedures and nonlinear response history analysis. The analytical models employed in this study are 2-dimensional models of two, four and eight-story archetype steel buildings designed as part of NIST [19]. Detailed 3-bay models and simplified 1-bay "models of the steel MRFs of these buildings are utilized, with the results being almost identical. Modeling of gravity framing can be achieved by means of a simple fishbone model. The main findings from the results presented here, which are representative for regular moment frames only, are summarized as follows:

- For regular frames of 4 and more stories, results from a single mode pushover analysis with an invariant load pattern do not correlate well with median results from nonlinear response history analysis. This holds true for story drifts, shear forces, and overturning moments.
- Modal Pushover Analysis leads to improved EDP predictions compared to the single mode NSP options by incorporating the second mode in the analysis in addition to the first mode. The second mode contribution is elastic for the cases evaluated in this study, which simplifies the modal combination.
- The sensitivity to invariant load patterns in single mode NSP and MPA typically leads to amplification of story drifts in lower stories compared to NRHA.
- Incorporating the effect of gravity system into the analytical model of the structural system typically leads to a reduction in story drift ratios compared to the bare frame only. This reduction may not be very important, except when the ground motion intensity is large and collapse becomes an issue. Further studies need to be conducted to address in detail the effect of gravity system on the seismic response of steel MRFs.
- Nonlinear static analysis procedures are not capable of providing relevant information on residual drifts and floor accelerations. These two EDPs are very important in loss assessment of buildings; the former for nonstructural acceleration sensitive damage, and the latter for assessing the need for demolition.
- In the authors' opinion, the main value of a nonlinear static (pushover) analysis is to inspect the load-deformation response at a global and local level for the purpose of evaluating behavior characteristics such as importance of P-Delta effect, global yielding, and post-yield and post-capping strength and stiffness characteristics, and for detection of potential strength and stiffness discontinuities that might adversely affect dynamic response. Quantification of demand parameters from pushover results is questionable for structures that have considerable higher mode effects and/or significant strength or stiffness discontinuities. Such quantification can be obtained, approximately, from NRHA using simple component models such a bilinear hysteresis model, and a small set of spectrum-matched ground motions. More accurate assessment of demand parameters, including measures of uncertainties, will necessitate more accurate structural modeling for NRHA and ground motions that represent the intensity characteristics and record-to-record variability inherent in seismic hazard.

ACKNOWLEDGEMENTS

This paper relies on results obtained under Task Order 6 of the NEHRP Consultants Joint Venture (a partnership of the Applied Technology Council and Consortium of Universities for Research in Earthquake Engineering), under Contract SB134107CQ0019, Earthquake Structural and Engineering Research, issued by the National Institute of Standards and Technology. The views expressed do not necessarily represent those of the organizations identified above.

REFERENCES

[1] ATC, "Seismic evaluation and retrofit of concrete buildings," ATC 40 Report, Volumes 1 and 2, Applied Technology Council, Redwood City, CA., 1996.

[2] ASCE, "Seismic evaluation of existing buildings," ASCE Standard ASCE/SEI 31-03, American Society of Civil Engineers/Structural Engineering Institute, Reston, VA., 2003.

[3] ASCE, "Seismic rehabilitation of existing buildings," ASCE Standard ASCE/SEI 41-06, American Society of Civil Engineers/Structural Engineering Institute, Reston, VA., 2007.

[4] FEMA, "NEHRP guidelines for the seismic rehabilitation of buildings," FEMA 273 Report, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C., 1997.

[5] FEMA, "Improvement of nonlinear static seismic analysis procedures," FEMA 440 Report, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C., 2005.

[6] FEMA, "Effects of strength and stiffness degradation on seismic response," FEMA P-440A Report, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C., 2009.

[7] FEMA, "Quantification of building seismic performance factors," FEMA P-695 Report, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C., 2009.

[8] M. Saiidi, M.A. Sozen, "Simple nonlinear seismic analysis of R/C structures," *Journal of the Structural Division,* ASCE*,* 107 (**ST5**), pp. 937–951, 1981.

[9] P. Fajfar P. Gaspersic, "The N2 method for the seismic damage analysis of RC buildings," *Earthquake Engineering and Structural Dynamics,* EESD*,* 25 **(1)**, pp. 31–46, 1996.

[10] R.S. Lawson, V. Vance, H. Krawinkler, "Nonlinear static pushover analysis — why, when and how?," *Proceedings 5th US Conf. Earthq. Engng* Chicago, IL, **(1)**, pp. 283–292, 1994.

[11] J.M. Bracci, S.K. Kunnath, A.M. Reinhorn, "Seismic performance and retrofit evaluation of reinforced concrete structures," *Journal of Structural Engineering,* ASCE*,* 123 **(1)**, pp. 3–10, 1997.

[12] H. Krawinkler, G.D.P.K. Seneviratna, "Pros and cons of a pushover analysis for seismic performance evaluation," *Journal of Engineering Structures*, 20 **(4-6)**, pp. 452-464, 1998.

[13] A.K. Chopra, R.K. Goel, "A modal pushover analysis procedure for estimating seismic demands for buildings," *Earthquake Engineering and Structural Dynamics*, EESD, 31 **(3)**, pp. 561-582, 2002.

[14] B. Gupta, S.K. Kunnath, "Adaptive spectra-based pushover procedure for seismic evaluation of structures," *Earthquake Spectra,* 16 **(2)**, pp. 367–392, 2000.

[15] K.K. Sasaki, S.A. Freeman, T.F. Paret, "Multimode pushover procedure (MMP)—a method to identify the effects of higher modes in a pushover analysis," *Proceedings of*

the 6th U.S. National Conference on Earthquake Engineering, Seattle, Washington, 1998.

[16] S. Antoniou, R. Pinho, "Development and verification of a displacement-based adaptive pushover procedure," *Journal of Earthquake Engineering*, 8 **(5)**, pp. 643-661, 2004.

[17] S. Antoniou, R. Pinho, "Advantages and limitations of adaptive and non-adaptive force-based pushover procedures," *Journal of Earthquake Engineering* 8 **(4)**, pp. 497-522, 2004.

[18] NIST, "Applicability of nonlinear multiple-degree-of-freedom modeling for design, GCR 10-917-9," prepared by the NEHRP Consultants Joint Venture for the National Institute of Standards and Technology, Gaithersburg, MD, 2010.

[19] NIST, "Evaluation of the FEMA P-695 methodology for quantification of building seismic performance factors, GCR 10-917-8," prepared by the NEHRP Consultants Joint Venture for the National Institute of Standards and Technology, Gaithersburg, MD, 2010.

[20] AISC, "Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications," AISC 358-05, American Institute of Steel Construction, Inc., Chicago, IL, 2005.

[21] D.G. Lignos, C. Putman, F. Zareian, H. Krawinkler, "Seismic evaluation of steel moment frames and shear walls using nonlinear static analysis procedures," *Proceedings ASCE Structures Congress*, Las Vegas, April 14th-16th, 2011.

[22] V. Prakash, G.H. Powell, S. Campbell, "DRAIN-2DX: Basic program description and user guide," *Report No. UCB/SEMM-1993/17*, University of California, Berkeley, CA, 1993.

[23] L.F. Ibarra, R.A. Medina, H. Krawinkler, "Hysteretic models that incorporate strength and stiffness deterioration," *Journal of Earthquake Engineering and Structural Dynamics*, EESD, 34 **(12)**, pp.1489–1511, 2005.

[24] D.G. Lignos, H. Krawinkler, "Deterioration modeling of steel components in support of collapse prediction of steel moment frames under earthquake loading," *Journal of Structural Engineering*, ASCE, (accepted for publication), 2011.

[25] H. Krawinkler, V.V. Bertero, E.P. Popov, "Shear behavior of steel frame joints," *Journal of the Structural Division,* ASCE, **101** (11), pp. 2317–2336, 1975.

[26] D.G. Lignos, F. Zareian, H. Krawinkler, "A Steel component database for deterioration modeling of steel beams with RBS under cyclic loading," *Proceedings ASCE Structures Congress,* Orlando Florida, May 12th-15th, 2010.

[27] N. Luco, Y. Mori, Y. Funahashi, A. Cornell, M. Nakashima, "Evaluation of predictors of non-linear seismic demands using 'fishbone' models of SMRF buildings," *Earthquake Engineering and Structural Dynamics*, EESD, 32 **(14)**, pp. 2267-2288, 2003.

[28] F. McKenna, "Object oriented finite element programming frameworks for analysis, algorithms and parallel computing," *Ph.D. Dissertation*, University of California, Berkeley, CA, 1997.

[29] PEER/ATC, "Modeling and acceptance criteria for seismic design and analysis of tall buildings," PEER/ATC-72-1, prepared by the Applied Technology Council in coopera-

tion with the Pacific Earthquake Engineering Research Center, Redwood City, CA, 2010.

[30] D.G. Lignos, "Interactive interface for incremental dynamic analysis, IIIDAP: Theory and example applications manual, Version 1.1.5," Department of Civil and Environmental Engineering, Stanford University, CA, 2009.

[31] A. Gupta, H. Krawinkler, "Dynamic P-Delta effects for flexible inelastic steel structures," *Journal of Structural Engineering*, ASCE, 126 **(1)**, pp. 145-154, 2000.

[32] J. Liu, A. Astaneh-Asl, "Cyclic testing of simple connections including effects of slab," *Journal of Structural Engineering*, ASCE, 126 **(1)**, pp. 32-39, 2000.

[33] J. Liu, A. Astaneh-Asl, ''Studies and tests of simple connections, including slab effects.'' *Report No. UCB/CEE-Steel-99/01*, University of California, Berkeley, CA, 1999.

[34] R. K. Goel, A. K. Chopra, "Extension of modal pushover analysis to compute member forces", *Earthquake Spectra*, 21, pp. 125-140, 2005.

[35] A.K. Chopra, "Dynamics of structures. Theory and applications to earthquake engineering", *3rd Edition*, Pearson Prentice Hall, Upper Saddle River, NJ, 2007.

[36] A. Gupta, H. Krawinkler. "Seismic demands for performance evaluation of steel moment resisting frame structures (SAC Task 5.4.3)," *Report No. 132*, John A. Blume Earthquake Engineering Center, Stanford University, Stanford, CA., 1997.

[37] S. Pampanin, C. Christopoulos, M.J.N. Priestley, "Residual deformations in the performance-seismic assessment of frame structures," *Report No. ROSE-2002/02*, European School for Advanced Studies in Reduction of Seismic Risk, Pavia, Italy, 2002.

[38] H. Aslani, E. Miranda, "Probabilistic earthquake loss estimation and loss disaggregation in buildings," *Report No. 157*, John A. Blume Earthquake Engineering Center, Stanford University, Stanford, CA, 2005.

[39] C.M. Ramirez, E. Miranda, "Building-specific loss estimation methods & tools for simplified performance-based earthquake engineering," *Report No. 173*, John A. Blume Earthquake Engineering Center, Stanford University, Stanford, CA, 2009.